Inlet Design

Q = CIA   Minor Storm Average Return Frequency ______

Location	Area, A	Runoff Coeficient, C	Time of Concen-tration	Intensity, I	Runoff, Q	By-Pass, Qb	Crossover, Qc	Total, Q	Paving Slope, S	Depth of Curb Flow	Inlet Type	Intercepted, Qi	Bypassed, Qb
	acres	constant	minutes	inch/hour	cfs	cfs	cfs	cfs	ft/tt	inches		c's	cfs

Figure 3-9 Inlet Design Computation Form

3.5 Storm Drains

3.5.1 Introduction

After the tentative location of inlets has been determined and the inlets sized, the next logical step is the computation of the rate of discharge to be carried by each drain pipe and the determination of the size and gradient of pipe required to carry this discharge. The procedure is carried out for each section of pipe starting at the most upstream inlet and proceeding downstream. It should be recognized that the rate of discharge to be carried by any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. In other words, the inlets are designed to assure that the full pipe capacity is utilized. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

For ordinary conditions, drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.

3.5.2 Design Criteria

The standard recommended maximum and minimum slopes for storm drains shall conform to the following criteria:

1. The maximum hydraulic gradient shall not produce a velocity that exceeds 20 feet per second.
2. The minimum desirable physical slope shall be 0.5 percent or the slope which will produce a velocity of 3.0 feet per second when the storm drain is flowing full, whichever is greater.

In order to determine if design flows can be accommodated by the storm drains system without causing flooding, or causing flows to exit the system at unacceptable locations, the designer shall determine *the hydraulic gradient.* The following design criteria shall be followed when determining the elevation along the hydraulic grade line (HGL):

- The hydraulic grade line shall be 0.75 feet below the intake lip of any affected inlet, any manhole cover, or any entering nonpressurized system.
- The energy grade line shall not rise above the intake lip of any affected inlet, any manhole cover or any entering nonpressurized system.

All storm drains should be designed such that velocities of flow will not be less than 3.0 feet per second at design flow, with a minimum slope of 0.5 percent. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system.

<u>Location and Alignment</u>

In new subdivisions the center of the street is reserved for storm drain system. When construction of a storm drain system is necessary in the older parts of the town, the location is determine by the City. No structures may be placed over a public storm drain system.

<u>Depth of Cover</u>

The desired depth of cover above a storm drain pipe shall be 2 to 3 feet, with 1.5 feet being the absolute minimum at an inlet location. Depth of cover greater than 3 feet shall be avoided due to the possibility of the storm drain blocking access of sanitary sewer service lines to the main sanitary sewer lines.

<u>Material and Joints</u>

Only reinforced concrete storm drain pipe shall be used within the City limits, unless approved by the Director of Public Works and Utilities. Construction of pipe and joint shall conform to the City of Lincoln Standard Specification.

<u>Bar Grates on End Sections</u>

An open pipe inlet from an open channel (similar to a culvert inlet) into a closed pipe storm drain shall be designed and constructed with flared end sections with a bar grate. No bar grate is required on the end section of a pipe outlet into an open channel unless directed by the Director of Public Works and Utilities.

3.5.3 Design Procedures

The design of storm drain systems is generally divided into the operations listed below. Supporting documentation shall be submitted with development plans for review:

1. The first step is the determination of inlet location and spacing as outlined earlier in this chapter.
2. The second step is the preparation of a plan layout of the storm drain system establishing the following design data:

   a. Location of storm drains.
   b. Direction of flow.
   c. Location of manholes.
   d. Location of existing facilities such as water, gas, or underground cables.

3. The design of the storm drain system is then accomplished by determining drainage areas, computing runoff by rational method, and computing the hydraulic capacity by Manning's equation.
4. The storm drain design computation sheet (Figure 3-12) shall be used to summarize the preliminary system design computations.
5. The hydraulic grade line computation from Figure 3-14 shall be used to determine the hydraulic gradient. The hydraulic grade line profile shall be provided on the storm drain system plans for the minor design storm.

3.5.4 Capacity

Storm drain capacity for reinforced concrete pipe can be determined using Figure 3-13. For non-standard applications, hydraulic capacity can be determined using the information provided below.

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning Formula and it is expressed by the following equation:

$$\mathbf{V = [1.486\ R^{2/3}S^{1/2}]/n} \quad \mathbf{(3.6)}$$

Where: V = mean velocity of flow (ft/s)
R = the hydraulic radius (ft) - the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
S = the slope of hydraulic grade line (ft/ft)
n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$\mathbf{Q = [1.486\ AR^{2/3}S^{1/2}]n} \quad \mathbf{(3.7)}$$

Where: Q = rate of flow (cfs)
A = cross sectional area of flow (ft^2)

For pipes flowing full, the above equations become:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}]/n} \quad \mathbf{(3.8)}$$
$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}]/n} \quad \mathbf{(3.9)}$$

Where: D = diameter of pipe (ft)

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = [2.87\ n^2V^2L]/[S^{4/3}]} \quad \mathbf{(3.10)}$$

$$H_f = [29\, n^2 L V^2]/[(R^{4/3})(2g)] \qquad (3.11)$$

Where:
H_f = total head loss due to friction (ft)
D = diameter of pipe (ft)
L = length of pipe (ft)
V = mean velocity (ft/s)
R = hydraulic radius (ft)
g = acceleration of gravity - 32.2 ft/s^2

3.5.4.1 Street Right-of-way and Overland Swale

Street right-of-ways convey the portion runoff in excess of pipe capacity, whether planned or not. Street right-of-way capacity is determined using Manning's equation for open channel flow conditions.

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \qquad (3.6)$$

The City of Lincoln uses standard street and right-of-way cross-sections for municipal streets, the formula can be simplified to:

$$Q = K\, S^{1/2}, \text{ where conveyance constant, } K = \frac{1.486}{n} A R^{2/3}$$

Area, wetted perimeter, and roughness coefficient are constant, the only variable being the street slope.

(i.e., For residential

A = 21.662 square feet,
R = 0.360 feet, and
$n_{(wtd)}$ = 0.026

For overland swales

A = 22.5 square feet,
R = 0.149 feet, and
n = 0.032)

The following table gives the conveyance constants for residential, commercial and major two-lane streets and a 30-foot wide swale with 10:1 side slopes.

Table 3-6 Conveyance Constants for Standard Street Right-of-Ways and 30' Swale

Residential	620
Business with parking	970
Business without parking	790
Major two-lane	1100
30-foot Swale	780

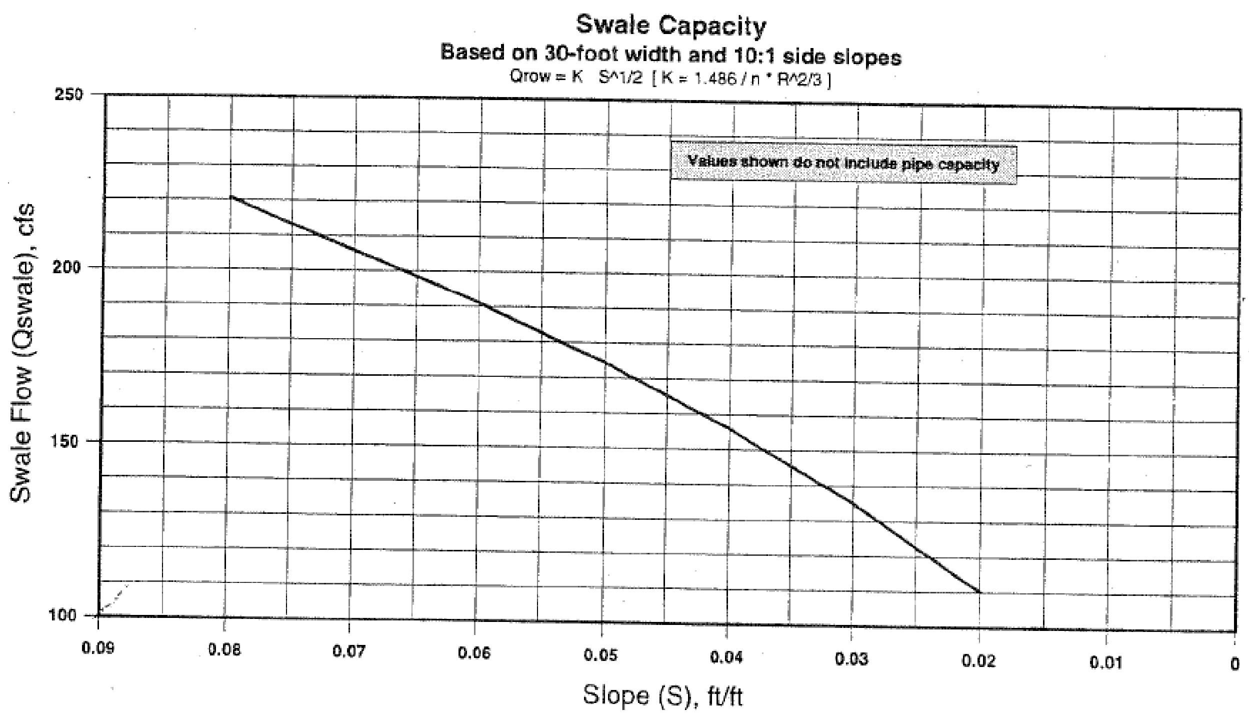


Figure 3-10 Swale Capacity Chart

TABLE III: Circular Concrete and Corrugated Metal Pipe

Pipe Dia. (Inch)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Values of $1.486/n \times A \times R^{2/3}$					
			Concrete Pipe			Corrugated Metal Pipe		
			n = 0.011	n = 0.012	n = 0.013	2-2/3" x 1/2" n = 0.024	3" x 1" n = 0.027	6" x 2" n = 0.033
8	0.349	0.167	14.3	13.1	12.1	6.5		
10	0.545	0.208	25.8	23.6	21.8	11.8		
12	0.785	0.250	42.1	38.6	35.7	19.3		
15	1.227	0.312	76.5	70.1	64.7	35.0		
18	1.767	0.375	124.2	113.8	105.1	56.9		
21	2.405	0.437	187.1	171.5	158.3	85.7		
24	3.142	0.500	267.4	245.1	226.2	122.5		
27	3.976	0.562	365.8	335.3	309.6	167.7		
30	4.909	0.625	484.7	444.3	410.1	222.2		
33	5.940	0.688	623.6	573.7	529.6	286.9		
36	7.069	0.750	788	722	666	361	321	
42	9.621	0.875	1189	1090	1006	545	484	
48	12.566	1.000	1698	1556	1436	778	692	
54	15.904	1.125	2325	2131	1967	1065	947	
60	19.635	1.250	3077	2821	2604	1410	1254	1026
66	23.758	1.375	3967	3636	3357	1818	1616	1323
72	28.274	1.500	5004	4587	4234	2293	2039	1668
78	33.183	1.625	6195	5679	5242	2839	2524	2065
84	38.485	1.750	7549	6920	6388	3460	3075	2517
90	44.179	1.875	9078	8321	7681		3698	3026
96	50.266	2.000	10776	9878	9119		4390	3592
102	56.745	2.125	12671	11615	10722			4224
108	63.617	2.250	14756	13526	12486			4919
114	70.882	2.375	17044	15624	14422			5682
120	78.540	2.500	19544	17915	16537			6515
126	86.590	2.625	22255	20397	18829			7417
132	95.030	2.750	25200	23104	21327			8401
138	103.870	2.875	28372	26009	24011			9459
144	113.100	3.000	31780	29133	26894			10594
150	122.720	3.125						11810
156	132.730	3.250						13115
162	143.140	3.375						14504
168	153.940	3.500						16160
174	165.130	3.625						17551
180	176.710	3.750						19212

Table 3-7 Values of $1.486/n \times A \times R^{2/3}$ for Circular Concrete and Corrugated Metal Pipe

Source: ACPA, Design Data 4, Hydraulic Capacity of Sewers, Table III

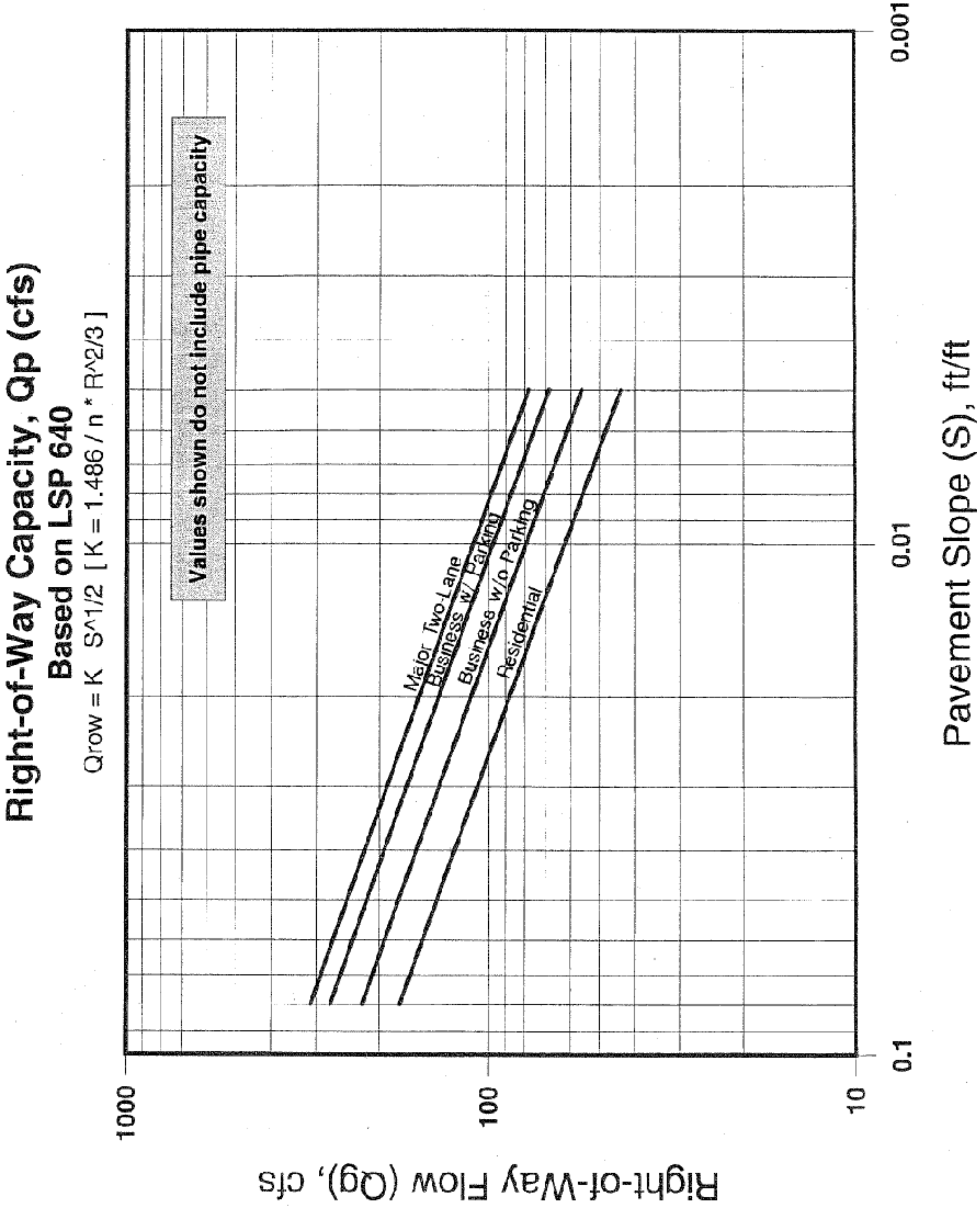


Figure 3-11 Right-of-Way Capacity Chart

Storm Drainage System

Column (1)	-	Contributing area at the point-of-study
Column (2)	-	Coefficient of runoff for Rational Method, see Table 2-3 and Table 2-4
Column (3)	-	Product of area and coefficient of runoff, C x A or Col. (3) = Col. (1) x Col. (2)
Column (4)	-	Summation of Col. (3) for all contributing drainage basins to the point-of-study
Column (5)	-	Time of concentration to the point of study for the drainage basin or the accumulated travel time of the aggregate drainage basins, whichever is greater, T_c
Column (6)	-	Minor storm rainfall intensity, from Figure 2-3 or $I = 42.456\ F^{0.1943} / (T_c+14.0)^{0.7912}$; F = Average Return Frequency
Column (7)	-	Peak rate of flow for minor storm runoff at the point-of-study, $Q_r = CIA$ or Col. (7) = Col. (4) x Col. (6)
Column (8)	-	Preliminary pipe slope
Column (9)	-	Pipe length segment from center to center of structures
Column (10)	-	Preliminary pipe size required to convey minor storm runoff. Indicate diameter or span x rise
Column (11)	-	Capacity of pipe for full flow conditions

$$Q = \frac{1.486}{0.013} A R^{2/3} S^{1/2} \text{ or Figure 3-13}$$

Column (12)	-	Velocity in the pipe for full-flow conditions, V = Q/A or Figure 3-13
Column (13)	-	Time of travel in pipe segment, $T_p = \frac{L}{60V}$ or Col. (13) = Col. (9) / Col. (12) / 60
Column (14)	-	100-year storm rainfall intensity, from Figure 2-3 or $I_{100} = 103.882 / (T_c + 14)^{0.7912}$
Column (15)	-	Peak rate of flow for 100-year storm runoff at the point-of-study, $Q_{100} = CI_{100}A$ or Col. (15) = Col. (4) x Col. (14)
Column (16)	-	Slope of overland flow route for 100-year storm runoff
Column (17)	-	Street and right-of-way width
Column (18)	-	Street capacity for flow to the limits of right-of-way for LSP-640

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$K = \frac{1.486}{n} A R^{2/3}$, is constant for full depth flow conditions.

K for each standard street and ROW width is provided below (e.g., K 26/60 for a 26' street with a 60-foot ROW)

Residential	$K_{(26/60)} = 620$	Business with parking	$K_{(38/72)} = 970$
Major two lane	$K_{(32/80)} = 1100$	Business without parking	$K_{(33/66)} = 790$
30-foot Swale	$K_{swale} = 780$		

or See Figure 3-10 or Figure 3-11

Column (19)	-	Combined capacity of the street and minor drainage systems must be equal to or greater than the peak rate of flow for the 100-year storm.
Column (20)	-	Swale width, where flow from major storms is not contained in the street system an overland flow route must be provided.
Column (21)	-	Combined capacity of the swale and minor systems must be equal to or greater than the peak rate of flow for the major storm.
Column (22)	-	Clarifying comments

Preliminary Pipe Sizing Calculations

Minor Storm System Conveyance Analysis

Minor Storm Average Return Frequency, years ________

Major Storm System Conveyance Analysis

Major Storm Average Return Frequency, 100 years

Location	Area, A acres	Coefficient, C	A * C	Sum A * C	Time of Concentration, Tc minutes	Intensity, I in/hour	Runoff, Qr cfs	Pipe Slope, Sp ft/ft	Pipe Length, L feet	Pipe Diameter, D inches	Pipe Capacity, Qp cfs	Pipe Velocity, Vp fps	Time in Section, Tp minutes	Intensity, I 100 in/hour	Flow, Q 100 cfs	Overflow Route Slope, Sov ft/ft	Street Width, Wst feet	Street Capacity, Qst cfs	Swale Width, W feet	Swale Capacity, Qsw cfs	Overflow plus Pipe Capacity, Qov cfs	Comments
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

(3) = (1) x (2) (6) $I_F = 42.456\ F^{0.1943} / (T + 14)^{0.7912}$ (7) $Q_F = C \times I_F \times A$ (11) $Q_P = (1.486 / n) \times A \times R^{2/3} \times S^{1/2}$ (12) $V_P = Q_P / A_P = (11) / A_F$

(13) $T_P = L / 60 / V_P$ (14) $Q_{100} = C \times I_{100} \times A$ (18) $Q_{ST} = K_{ST} \times S_{OV}$ (20) $Q_{SW} = K_{SW} \times S_{OV}$ (21) $Q_{OV} = (11) + (18) + (20)$

Figure 3-12 Storm Drain Computation Form

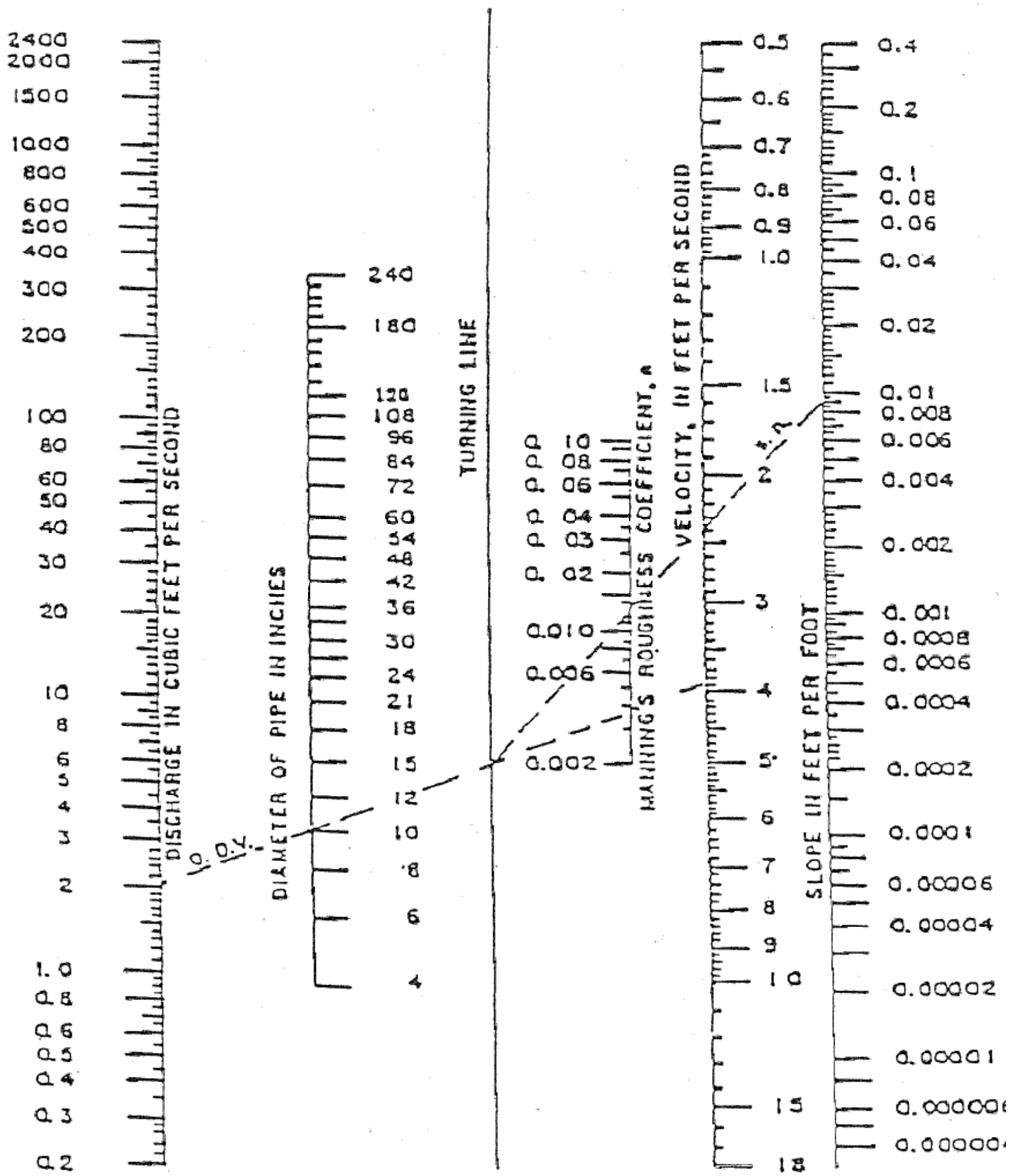


Figure 3-13 Nomograph For Solution of Manning's Formula In Storm Drains

3.5.5 Hydraulic Gradient

In order to determine if design flows can be accommodated by the storm drains system without causing flooding, or causing flows to exit the system at unacceptable locations, the designer shall determine the hydraulic gradient. Computing the hydraulic gradient will determine the elevation to which water will rise in inlets and manholes. The following sections provide the necessary procedures and equations to determine the hydraulic gradient.

3.5.5.1 Friction Losses

Energy losses from pipe friction may be determined by rewriting the Manning equation.

$$\mathbf{S_f = [Qn/1.486\ A(R^{2/3})]^2} \qquad \mathbf{(3.12)}$$

Then the head losses due to friction may be determined by the formula:

$$\mathbf{H_f = S_f L} \qquad \mathbf{(3.13)}$$

Where: H_f = friction head loss (ft)
S_f = friction slope (ft/ft)
L = length of outflow pipe (ft)

3.5.5.2 Velocity Head Losses

From the time storm water first enters the sewer system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as inlets, manholes, junctions, bends, contractions, enlargements and transitions, which will cause velocity head losses. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisback equations.

$$\mathbf{H = KV^2/2g} \qquad \mathbf{(3.14)}$$

Where: H = velocity head loss (ft)
K = loss coefficient for the particular structure
V = velocity of flow (ft/s)
g = acceleration due to gravity (32.2 ft/s)

3.5.5.3 Entrance Losses

Following are the equations used for entrance losses.

$$\mathbf{H_{tm} = V^2/2g} \qquad \mathbf{(3.15)}$$

$$\mathbf{H_e = KV^2/2g} \qquad \mathbf{(3.16)}$$

Where: H_{tm} = terminal (beginning of run) loss (ft)
H_e = entrance loss for outlet structure (ft)
K = 0.5 (assuming square-edge)
(Other terms defined above.)

3.5.5.4 Junction Losses

Incoming Opposing Flows

The head loss at a junction, H_{j1} for two almost equal and opposing flows meeting head on with the outlet direction perpendicular to both incoming directions, head loss is considered as the total velocity head of outgoing flow.

$$\mathbf{H_{j1} = (V^2)/2g} \qquad \mathbf{(3.17)}$$

Where: H_{j1} = junction losses (ft)
(Other terms are defined above.)

Changes in Direction of Flow

When main storm drain pipes or lateral lines meet in a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90°) the more severe this energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$\mathbf{H_b = K_b(V^2)/2g} \qquad \mathbf{(3.18)}$$

Where: H_b = bend head loss (ft)
K_b = junction loss coefficient

The following Table 3-8 lists the values of K_b for various changes in flow direction and junction angles.

Table 3-8 Values Of K_b For Change In Direction Of Flow In Lateral

K	Degree of Turn (In Junction)
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater

K values for other degree of turns can be obtained by interpolating between values.

Table 3-9 lists the values for the junction loss coefficient for various conditions at pipe junctions.

Table 3-9 Values Of K At Junctions

For no bends at junctions -	K = 0.20
For bends at junctions of 25 degrees -	K = 0.30
For bends at junctions of 45 degrees -	K = 0.40
For bends at junctions of 90 degrees -	K = 0.60
For junctions of three pipes -	K = 0.80
For junctions of four or more pipes -	K = 1.00

Several Entering Flows

The computation of losses in a junction with several entering flows utilizes the principle of conservation of energy. For a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction. The total junction losses can be determined from equation 3-17. See also Figure 3-14.

$$\mathbf{H_{j2} = [(Q_4V_4^2)-(Q_1V_1^2)-(Q_2V_2^2)+(KQ_1V_1^2)]/(2gQ_4)} \qquad \mathbf{(3.19)}$$

Where: H_{j2} = junction losses (ft)
Q = discharges (cfs)
V = horizontal velocities (ft/s) (V_3 is assumed to be zero)
g = acceleration due to gravity (32.2 ft/s^2)
K = bend loss factor

Where subscript nomenclature is as follows:

Q_1 = 90º lateral (cfs)
Q_2 = straight through inflow (cfs)
Q_3 = vertical dropped-in flow from an inlet (cfs)
Q_4 = main outfall = total computed discharge (cfs)
V_1,V_2,V_3,V_4 are the horizontal velocities of foregoing flows, respectively, in feet per second
V_3 assumed to be = 0

Also Assume:

- $H_b = K(V_1^2)/2g$ for change in direction.
- No velocity head of an incoming line is greater than the velocity head of the outgoing line.
- Water surface of inflow and outflow pipes in junction to be level.

When losses are computed for any junction condition for the same or a lesser number of inflows, the above equation will be used with zero quantities for those conditions not present. If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

3.5.5.5 Summary

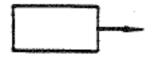
The final step in designing a storm drain system is to check the hydraulic grade line (HGL) as described in the next section of this chapter. Computing the HGL will determine the elevation, under design conditions, to which water will rise in various inlets, manholes, junctions, and etc. The following design criteria shall be followed when determining the elevation at the HGL:

- The hydraulic grade line shall be 0.75 feet below the intake lip of any affected inlet, any manhole cover, or any entering nonpressurized system.
- The energy grade line shall not rise above the intake lip of any affected inlet, any manhole cover or any entering nonpressurized system.

A summary of energy losses which shall be considered is presented in Figure 3-14.

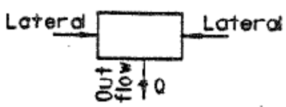
$$H_{tm} = \frac{v^2}{2g}$$

TERMINAL LOSSES
(at beginning of run)
Where g = gravitational constant. 32.2 feet per second per second.



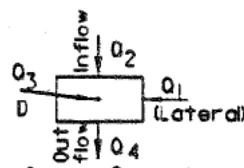
$$H_e = \frac{0.5\,v^2}{2g}$$

ENTRANCE LOSSES
(at end of run)
Assuming square - edge



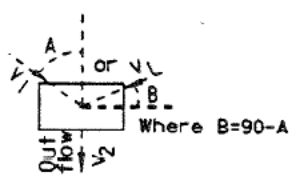
$$H_{J1} = \frac{v^2 (\text{Outflow})}{2g}$$

JUNCTION LOSSES
(Incoming-opposing Flow)
Use only where flows are identical to above, otherwise use H_{J2} Equation.



$$H_{J2} = \frac{Q_4 V_4^2 - Q_1 V_1^2 - Q_2 V_2^2 + K Q_1 V_1^2}{2 g Q_4}$$

JUNCTION LOSSES
(Several Entering Flows)
Total losses to include H_{J2} plus losses for changes in direction of less than 90° (h_b).
Where K = Bend loss coefficient
Q_3 = Vertical dropped-in flow from an inlet
V_3 = Assumed to be zero



$$H_b = \frac{K V_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSSES (H_f)

$H_f = S_f \times L$

Where H_f = friction head loss
S_f = friction slope
L = length of conduit

$$\text{Where } S = \left(\frac{Qn}{1.486 A R^{\frac{2}{3}}}\right)^2$$

Q = Discharge of conduit
n = Mannings coefficient of roughness
A = area of conduit
R = hydraulic radius of conduit

TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + H_e + (H_{J1} \text{ or } H_{J2}) + H_b + H_f$$

Figure 3-14 Summary Of Energy Losses

Source: AASHTO Model Drainage Manual, 1991

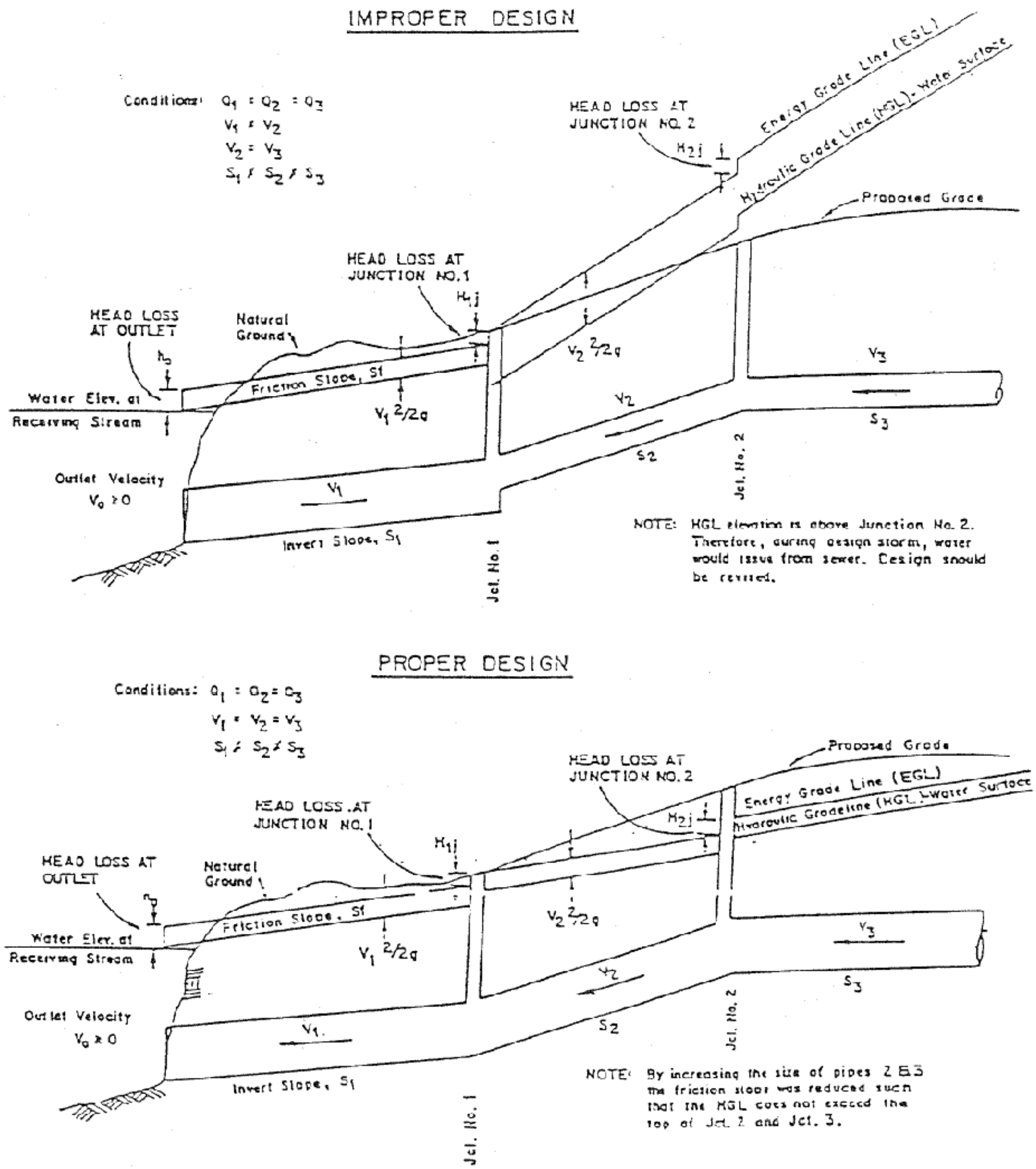


Figure 3-15 Energy And Hydraulic Grade Lines For Storm drain Under Constant Discharge

Source: AASHTO Model Drainage Manual, 1991

3.5.6 Hydraulic Grade Line Design Procedure

The hydraulic grade line is calculated beginning at the system outlet proceeding upstream. Conditions expected at the outlet for the minor design storm shall be used for the starting water surface elevation.

Column (1) - Design flow to be conveyed by pipe segment.
Column (2) - Length of pipe segment.
Column (3) - Pipe Size; Indicate pipe diameter or span x rise.
Column (4) - Constant K, from American Concrete Pipe Association Design Data:

$$Kp = \frac{1{,}486}{n} AR^{2/3} \text{ ; or from Table 3-7}$$

Column (5) - Flowline Outlet Elevation of pipe segment.
Column (6) - Flowline Inlet Elevation of pipe segment.
Column (7) - Barrel Area is the full cross sectional area of the pipe.
Column (8) - Barrel Velocity is the full velocity in the pipe as determined by:

$V = Q/A$ or Col. (8) = Col. (1) / Col. (7)

Column (9) - Barrel Velocity Head = $V^2/2g$ or Col. $(8)^2 / 2g$
Where, $g = 32.2$ ft/sec^2 (acceleration due to gravity)

Column (10) - Tailwater (TW) Elevation; this is the water surface elevation at the outlet of the pipe segment. If the pipe's outlet is not submerged by the TW and the TW depth is less than $(D+d_c)/2$, set the TW elevation equal to $(D+d_c)/2$. This will keep the analysis simple yet still obtain reasonable results (D = pipe barrel height and d_c = critical depth, both in ft. See Appendix 4-B for determination of d_c).

Column (11) - Friction Loss = S_1 x L or S_1 x Col. (2)
Where, S_1 is the friction slope or head loss per lineal foot of pipe as determined by Manning's Equation expressed in the form:

$S_1 = S_f = (Q / K_p)^2$; K from Table 3-7

Column (12) - Hydraulic Grade Line (HGL) Elevation just inside the entrance of the pipe barrel; this is determined by adding the friction loss to the TW elevation:

Col. (12) = Col. (11) + Col. (10)

If this elevation falls below the pipe's inlet crown, it no longer represents the true HGL when computed in this manner. The true HGL will fall somewhere between the pipe's crown and either normal flow depth or critical flow depth, whichever is greater. To keep the analysis simple and still obtain reasonable results (i.e., erring on the conservative side), set the HGL elevation equal to the crown elevation.

Column (13) - Entrance Head Loss = K_e x $V^2 / 2g$ or K_e x Col. (9) Where, K_e = Entrance Loss Coefficient (0.5 assuming square-edge) This is the head lost due to flow contractions at the pipe entrance.

Column (14) - Exit Head Loss = 1.0 x $V^2 / 2g$ or 1.0 x Col. (9)
This is the velocity head lost or transferred downstream.

Column (15) - Outlet Control Elevation = Col. (12) + Col. (13) + Col. (14)
This is the maximum headwater elevation assuming the pipe's barrel and inlet/outlet characteristics are controlling capacity. It does not include structure losses or approach velocity considerations.

Column (16) - Inlet Control Elevation (See Figure 4-2 for computation of inlet control on culverts). This is the maximum headwater elevation assuming the pipe's inlet is controlling capacity. It does not include structure losses or approach velocity considerations.

Column (17) - Approach Velocity Head; this is the head (energy) being supplied by the discharge form an upstream pipe or channel section, which serves to reduce the headwater elevation. If the discharge is from a pipe, the approach velocity head is equal to the barrel velocity head computed for the upstream pipe. If the upstream pipe outlet is significantly higher in elevation (as in a drop manhole) or lower in elevation such that its discharge energy would be dissipated, an approach velocity head of zero should be assumed.

Column (18) - Bend Head Loss = K_b x V^2 / 2g or k_b x Col. (17)
Where, K_b = Bend Loss Coefficient (from Table 3-7). This is the loss of head/energy required to change direction of flow in an access structure.

Column (19) - Junction Head Loss; this is the loss in head (energy) that results from the turbulence created when two or more streams are merged into one within the access structure. Table 3-8 can be used to determine junction loss coefficients for use in the following equations given in Figure 3-14.

Column (20) - Headwater (HW) Elevation; this is determined by combining the energy heads in Columns 17, 18, and 19 with the highest control elevation in either Column 15 or 16, as follows:

Col. (20) = Col. (15 or 16) - Col. (17) + Col. (18) + Col. (19)

Column (21) - Top of curb elevation at an inlet or rim elevation at a storm sewer manhole.

Column (22) - Inlet capacity is reduced if the hydraulic gradeline elevation interferes with the napping effect during weir or orifice flow conditions.

Hydraulic Grade Line Calculation Sheet

Minor Storm Ave. Return Frequency ____

Pipe Segment		Q (cfs)	Length (feet)	Pipe Siz e (in)	Ko	Outlet Elevation (feet)	Inlet Elevation (feet)	Barrel Area (sqft)	Barrel Velocity (fps)	Barrel Velocity Head (feet)	Tail Water Elevation (feet)	Fric-tion Loss (feet)	Entrance HGL Elevation (feet)	Entrance Head Loss (feet)	Exit Head Loss (feet)	Outlet Control Elevation (feet)	Inlet Control Elevation (feet)	Appr. Vel. Head (feet)	Bend Head Loss (feet)	Junc. Head Loss (feet)	HW Elevation (feet)	Top Curb Elevation (feet)	Elevation Difference (ft)	Comment
from	to	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)

(8) = Q/A = (1)/(7) (9) = $h_v = V^2/2g$ = (8)/64.4 (11) = h_f = SL; S_f=Q/K; K from Table 3-7 (12) 10 + 11

(13) = $K_e(V^2/2g) = K_e(9)$ (14) = $1.0(V^2/2g) = 1.0(9)$ (15) 12+13+14 (16) Figure 4-2 (17) = (9)

(18) = $K_b(V^2/2g) = K_b(17)$ (19) = $K_j(V^2/2g) = K_j(17)$ (20) = (15 or 16)-(17)+(18)+(19) (22) Reduces Inlet Capacity if < 1.5 feet

Figure 3-16 Hydraulic Grade Line Computation Form

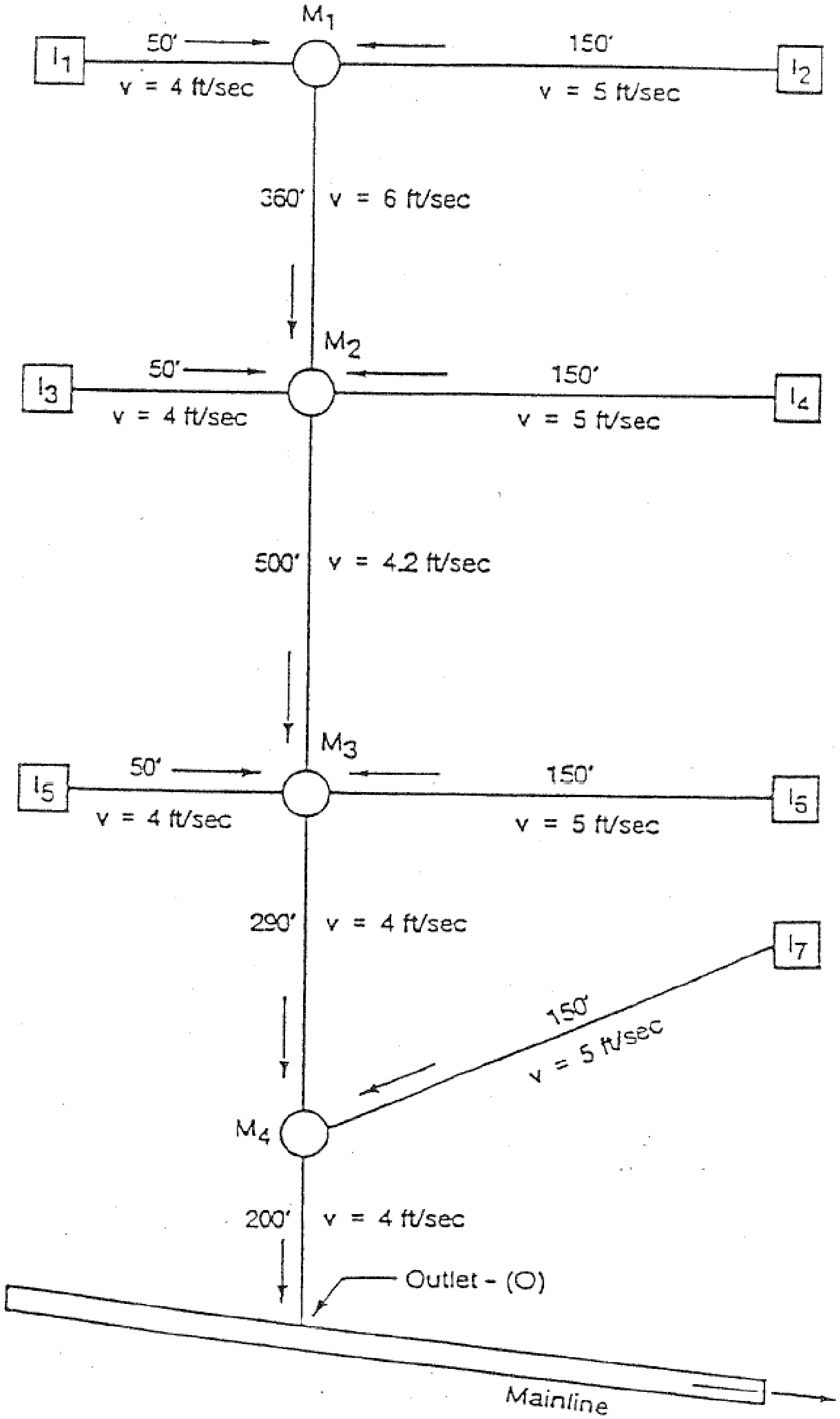


Figure 3-17 Hypothetical Storm Drain System Layout

3.6 Computer Programs

There are numerous proprietary and non-proprietary computer models that may be used to design components of the minor storm drainage system. The reader is referred to the user manual for any particular program to determine its suitability for solving storm drainage problems.

References

U. S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12.

American Concrete Pipe Association, March 1968, Design Data.